APPENDIX

AMERICAN WOOD COUNCIL

Appendix A (Non-mandatory) Construction and Design Practices

A.1 Care of Material

Lumber shall be so handled and covered as to prevent marring and moisture absorption from snow or rain.

A.2 Foundations

A.2.1 Foundations shall be adequate to support the building or structure and any required loads, without excessive or unequal settlement or uplift.

A.2.2 Good construction practices generally eliminate decay or termite damage. Such practices are designed to prevent conditions which would be conducive to decay and insect attack. The building site shall be graded to provide drainage away from the structure. All roots and scraps of lumber shall be removed from the immediate vicinity of the building before backfilling.

A.3 Structural Design

Consideration shall be given in design to the possible effect of cross-grain dimensional changes which may occur in lumber fabricated or erected in a green condition (i.e., provisions shall be made in the design so that if dimensional changes caused by seasoning to moisture equilibrium occur, the structure will move as a whole, and the differential movement of similar parts and members meeting at connections will be a minimum).

A.4 Drainage

In exterior structures, the design shall be such as to minimize pockets in which moisture can accumulate, or adequate caps, drainage, and drips shall be provided.

A.5 Camber

Adequate camber in trusses to give proper appearance and to counteract any deflection from loading should be provided. For timber connector construction, such camber shall be permitted to be estimated from the formula:

$$\Delta = \frac{K_1 L^3 + K_2 L^2}{H} \quad \text{(A-1)}$$

where:

Δ = camber at center of truss, in.

L = truss span, ft

H = truss height at center, ft

K_1 = 0.000032 for any type of truss

K_2 = 0.0028 for flat and pitched trusses

K_2 = 0.00063 for bowstring trusses (i.e., trusses without splices in upper chord)

A.6 Erection

A.6.1 Provision shall be made to prevent the overstressing of members or connections during erection.

A.6.2 Bolted connections shall be snugly tightened, but not to the extent of crushing wood under washers.

A.6.3 Adequate bracing shall be provided until permanent bracing and/or diaphragms are installed.

A.7 Inspection

Provision should be made for competent inspection of materials and workmanship.

A.8 Maintenance

There shall be competent inspection and tightening of bolts in connections of trusses and structural frames.

A.9 Wood Column Bracing

In buildings, for forces acting in a direction parallel to the truss or beam, column bracing shall be permitted to be provided by knee braces or, in the case of trusses, by extending the column to the top chord of the truss where the bottom and top chords are separated sufficiently to provide adequate bracing action. In a direction perpendicular to the truss or beam, bracing shall be permitted to be provided by wall construction, knee braces, or bracing between columns. Such bracing between columns should be installed preferably in the same bays as the bracing between trusses.

A.10 Truss Bracing

In buildings, truss bracing to resist lateral forces shall be permitted as follows:

(a) Diagonal lateral bracing between top chords of trusses shall be permitted to be omitted when the provisions of Appendix A.11 are followed or when the roof joists rest on and are securely fastened to the top chords of the trusses and are covered with wood sheathing. Where sheathing other than wood is applied, top chord diagonal lateral bracing should be installed.

(b) In all cases, vertical sway bracing should be installed in each third or fourth bay at intervals of approximately 35 feet measured parallel to trusses. Also, bottom chord lateral bracing should be installed in the same bays as the vertical sway bracing, where practical, and should extend from side wall to side wall. In addition, struts should be installed between bottom chords at the same truss panels as vertical sway bracing and should extend continuously from end wall to end wall. If the roof construction does not provide proper top chord strut action, separate additional members should be provided.

A.11 Lateral Support of Arches, Compression Chords of Trusses and Studs

A.11.1 When roof joists or purlins are used between arches or compression chords, or when roof joists or purlins are placed on top of an arch or compression chord, and are securely fastened to the arch or compression chord, the largest value of ℓ_e/d, calculated using the depth of the arch or compression chord or calculated using the breadth (least dimension) of the arch or compression chord between points of intermittent lateral support, shall be used. The roof joists or purlins should be placed to account for shrinkage (for example by placing the upper edges of unseasoned joists approximately 5% of the joist depth above the tops of the arch or chord), but also placed low enough to provide adequate lateral support.

A.11.2 When planks are placed on top of an arch or compression chord, and securely fastened to the arch or compression chord, or when sheathing is nailed properly to the top chord of trussed rafters, the depth rather than the breadth of the arch, compression chord, or trussed rafter shall be permitted to be used as the least dimension in determining ℓ_e/d.

A.11.3 When stud walls in light frame construction are adequately sheathed on at least one side, the depth, rather than breadth of the stud, shall be permitted to be taken as the least dimension in calculating the ℓ_e/d ratio. The sheathing shall be shown by experience to provide lateral support and shall be adequately fastened.

Appendix B (Non-mandatory) Load Duration (ASD Only)

B.1 Adjustment of Reference Design Values for Load Duration

B.1.1 Normal Load Duration. The reference design values in this Specification are for normal load duration. Normal load duration contemplates fully stressing a member to its allowable design value by the application of the full design load for a cumulative duration of approximately 10 years and/or the application of 90% of the full design load continuously throughout the remainder of the life of the structure, without encroaching on the factor of safety.

B.1.2 Other Load Durations. Since tests have shown that wood has the property of carrying substantially greater maximum loads for short durations than for long durations of loading, reference design values for normal load duration shall be multiplied by load duration factors, C_D, for other durations of load (see Figure B1). Load duration factors do not apply to reference modulus of elasticity design values, E, nor to reference compression design values perpendicular to grain, $F_{c\perp}$, based on a deformation limit.

(a) When the member is fully stressed to the adjusted design value by application of the full design load permanently, or for a cumulative total of more than 10 years, reference design values for normal load duration (except E and $F_{c\perp}$ based on a deformation limit) shall be multiplied by the load duration factor, $C_D = 0.90$.

(b) Likewise, when the duration of the full design load does not exceed the following durations, reference design values for normal load duration (except E and $F_{c\perp}$ based on a deformation limit) shall be multiplied by the following load duration factors:

C_D	Load Duration
1.15	two months duration
1.25	seven days duration
1.6	ten minutes duration
2.0	impact

(c) The 2 month load duration factor, $C_D = 1.15$, is applicable to design snow loads based on ASCE 7. Other load duration factors shall be permitted to be used where such adjustments are referenced to the duration of the design snow load in the specific location being considered.

(d) The 10 minutes load duration factor, $C_D = 1.6$, is applicable to design earthquake loads and design wind loads based on ASCE 7.

(e) Load duration factors greater than 1.6 shall not apply to structural members pressure-treated with water-borne preservatives (see Reference 30), or fire retardant chemicals. The impact load duration factor shall not apply to connections.

B.2 Combinations of Loads of Different Durations

When loads of different durations are applied simultaneously to members which have full lateral support to prevent buckling, the design of structural members and connections shall be based on the critical load combination determined from the following procedures:

(a) Determine the magnitude of each load that will occur on a structural member and accumulate subtotals of combinations of these loads. Design loads established by applicable building codes and standards may include load combination factors to adjust for probability of simultaneous occurrence of various loads (see Appendix B.4). Such load combination factors should be included in the load combination subtotals.

(b) Divide each subtotal by the load duration factor, C_D, for the shortest duration load in the combination of loads under consideration.

Shortest Load Duration in the Combination of Loads	Load Duration Factor, C_D
Permanent	0.9
Normal	1.0
Two Months	1.15
Seven Days	1.25
Ten Minutes	1.6
Impact	2.0

(c) The largest value thus obtained indicates the critical load combination to be used in designing the structural member or connection.

EXAMPLE: Determine the critical load combination for a structural member subjected to the following loads established by applicable building code or standard:

D = dead load
L = live load

S = snow load
W = wind load

The actual stress due to any combination of the above loads shall be less than or equal to the adjusted design value modified by the load duration factor, C_D, for the shortest duration load in that combination of loads:

Actual stress due to	(C_D)	x (Design value)
D	≤ (0.9)	x (design value)
D+L	≤ (1.0)	x (design value)
D+W	≤ (1.6)	x (design value)
D+L+S	≤ (1.15)	x (design value)
D+L+W	≤ (1.6)	x (design value)
D+S+W	≤ (1.6)	x (design value)
D+L+S+W	≤ (1.6)	x (design value)

The equations above may be specified by the applicable building code and shall be checked as required. Load combination factors specified by the applicable building code or standard should be included in the above equations, as specified in B.2(a).

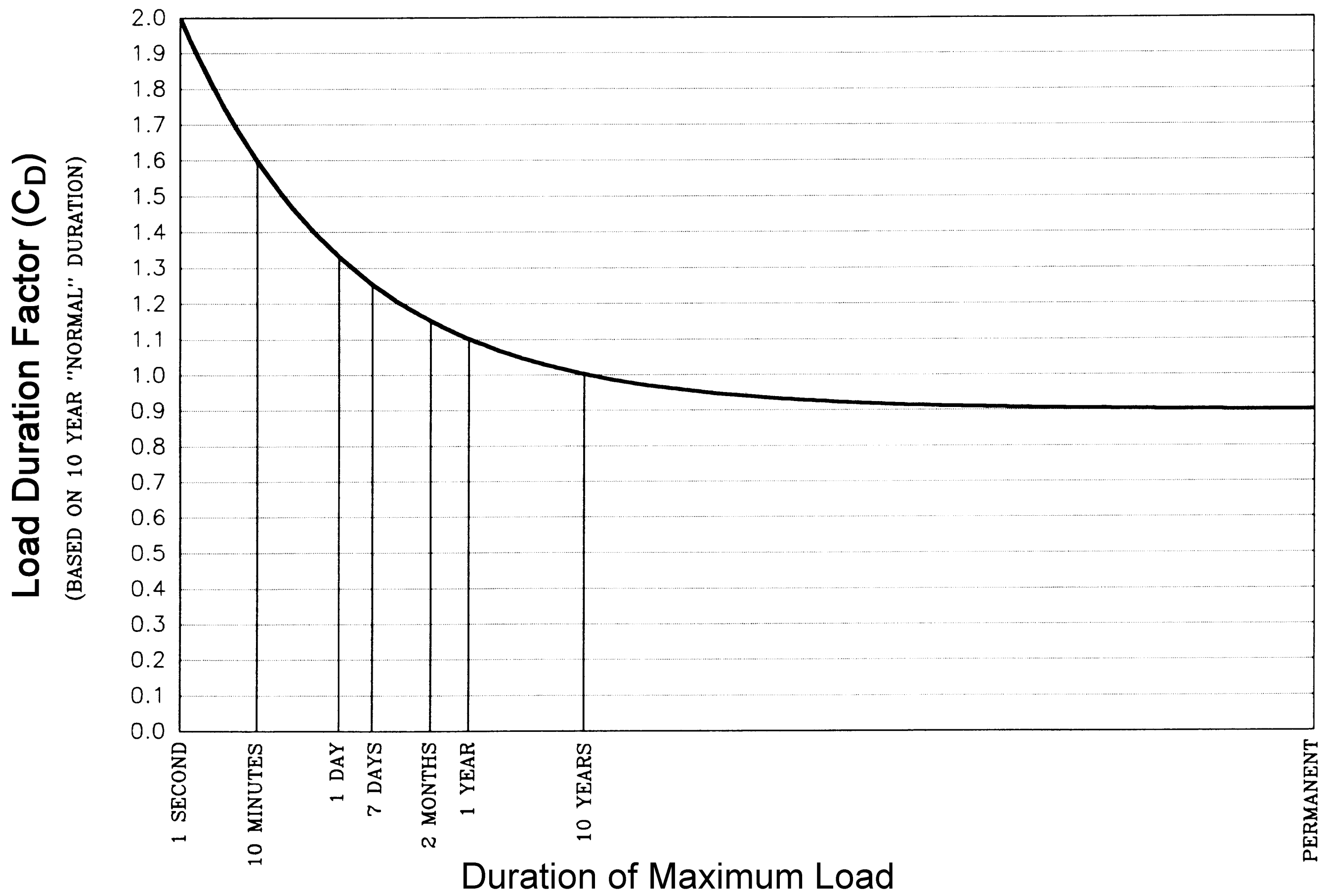
B.3 Mechanical Connections

Load duration factors, $C_D \leq 1.6$, apply to reference design values for connections, except when connection capacity is based on design of metal parts (see 11.2.3).

B.4 Load Combination Reduction Factors

Reductions in total design load for certain combinations of loads account for the reduced probability of simultaneous occurrence of the various design loads. Load duration factors, C_D, account for the relationship between wood strength and time under load. Load duration factors, C_D, are independent of load combination reduction factors, and both may be used in design calculations (see 1.4.4).

Figure B1 Load Duration Factors, C_D, for Various Load Durations



Appendix C (Non-mandatory) Temperature Effects

C.1

As wood is cooled below normal temperatures, its strength increases. When heated, its strength decreases. This temperature effect is immediate and its magnitude varies depending on the moisture content of the wood. Up to 150°F, the immediate effect is reversible. The member will recover essentially all its strength when the temperature is reduced to normal. Prolonged heating to temperatures above 150°F can cause a permanent loss of strength.

C.2

In some regions, structural members are periodically exposed to fairly elevated temperatures. However, the normal accompanying relative humidity generally is very low and, as a result, wood moisture contents also are low. The immediate effect of the periodic exposure to the elevated temperatures is less pronounced because of this dryness. Also, independently of temperature changes, wood strength properties generally increase with a decrease in moisture content. In recognition of these offsetting factors, it is traditional practice to use the reference design values from this Specification for ordinary temperature fluctuations and occasional short-term heating to temperatures up to 150°F.

C.3

When wood structural members are heated to temperatures up to 150°F for extended periods of time, adjustment of the reference design values in this Specification may be necessary (see 2.3.3 and 11.3.4). See Reference 53 for additional information concerning the effect of temperature on wood strength.

Appendix D (Non-mandatory) Lateral Stability of Beams

D.1

Slenderness ratios and related equations for adjusting reference bending design values for lateral buckling in 3.3.3 are based on theoretical analyses and beam verification tests.

D.2

Treatment of lateral buckling in beams parallels that for columns given in 3.7.1 and Appendix H. Beam stability calculations are based on slenderness ratio, R_B, defined as:

$$R_B = \sqrt{\frac{\ell_e d}{b^2}} \qquad \text{(D-1)}$$

with ℓ_e as specified in 3.3.3.

D.3

For beams with rectangular cross section where R_B does not exceed 50, adjusted bending design values are obtained by the equation (where $C_L \leq C_V$):

$$F_b' = F_b^* \left[\frac{1+\left(F_{bE}/F_b^*\right)}{1.9} - \sqrt{\left[\frac{1+\left(F_{bE}/F_b^*\right)}{1.9}\right]^2 - \frac{F_{bE}/F_b^*}{0.95}} \right] \qquad \text{(D-2)}$$

where:

$$F_{bE} = \frac{1.20\, E_{min}'}{R_B^{\,2}} \qquad \text{(D-3)}$$

F_b^* = reference bending design value multiplied by all applicable adjustment factors except C_{fu}, C_V (when $C_V \leq 1.0$), and C_L (see 2.3)

D.4

Reference modulus of elasticity for beam and column stability, E_{min}, in Equation D-3 is based on the following equation:

$$E_{min} = E\,[1 - 1.645\,COV_E](1.03)/1.66 \qquad \text{(D-4)}$$

where:

E = reference modulus of elasticity

1.03 = adjustment factor to convert E values to a pure bending basis except that the factor is 1.05 for structural glued laminated timber

1.66 = factor of safety

COV_E = coefficient of variation in modulus of elasticity (see Appendix F)

E_{min} represents an approximate 5% lower exclusion value on pure bending modulus of elasticity, plus a 1.66 factor of safety.

D.5

For products with less E variability than visually graded sawn lumber, higher critical buckling design values (F_{bE}) may be calculated. For a product having a lower coefficient of variation in modulus of elasticity, use of Equations D-3 and D-4 will provide a 1.66 factor of safety at the 5% lower exclusion value.

Appendix E (Non-mandatory) Local Stresses in Fastener Groups

E.1 General

Where a fastener group is composed of closely spaced fasteners loaded parallel to grain, the capacity of the fastener group may be limited by wood failure at the net section or tear-out around the fasteners caused by local stresses. One method to evaluate member strength for local stresses around fastener groups is outlined in the following procedures.

E.1.1 Reference design values for timber rivet connections in Chapter 14 account for local stress effects and do not require further modification by procedures outlined in this Appendix.

E.1.2 The capacity of connections with closely spaced, large diameter bolts has been shown to be limited by the capacity of the wood surrounding the connection. Connections with groups of smaller diameter fasteners, such as typical nailed connections in wood-frame construction, may not be limited by wood capacity.

E.2 Net Section Tension Capacity

The adjusted tension capacity is calculated in accordance with provisions of 3.1.2 and 3.8.1 as follows:

$$Z_{NT}' = F_t' A_{net} \tag{E.2-1}$$

where:

Z_{NT}' = adjusted tension capacity of net section area

F_t' = adjusted tension design value parallel to grain

A_{net} = net section area per 3.1.2

E.3 Row Tear-Out Capacity

The adjusted tear-out capacity of a row of fasteners can be estimated as follows:

$$Z_{RTi}' = n_i \frac{F_v' A_{critical}}{2} \tag{E.3-1}$$

where:

Z_{RTi}' = adjusted row tear out capacity of row i

F_v' = adjusted shear design value parallel to grain

$A_{critical}$ = minimum shear area of any fastener in row i

n_i = number of fasteners in row i

E3.1 Assuming one shear line on each side of bolts in a row (observed in tests of bolted connections), Equation E.3-1 becomes:

$$Z_{RTi}' = \frac{F_v' t}{2}\left[n_i s_{critical}\right]\left(2 \text{ shear lines}\right) \tag{E.3-2}$$

$$= n_i F_v' t s_{critical}$$

where:

$s_{critical}$ = minimum spacing in row i taken as the lesser of the end distance or the spacing between fasteners in row i

t = thickness of member

The total adjusted row tear-out capacity of multiple rows of fasteners can be estimated as:

$$Z_{RT}' = \sum_{i=1}^{n_{row}} Z_{RTi}' \tag{E.3-3}$$

where:

Z_{RT}' = adjusted row tear out capacity of multiple rows

n_{row} = number of rows

E.3.2 In Equation E.3-1, it is assumed that the induced shear stress varies from a maximum value of $f_v = F_v'$ to a minimum value of $f_v = 0$ along each shear line between fasteners in a row and that the change in shear stress/strain is linear along each shear line. The resulting triangular stress distribution on each shear line between fasteners in a row establishes an apparent shear stress equal to half of the adjusted design shear stress, $F_v'/2$, as shown in Equation E.3-1. This assumption is combined with the critical area concept for evaluating stresses in fastener groups and provides good agreement with results from tests of bolted connections.

E3.3 Use of the minimum shear area of any fastener in a row for calculation of row tear-out capacity is based on the assumption that the smallest shear area between fasteners in a row will limit the capacity of the row of fasteners. Limited verification of this approach is provided from tests of bolted connections.

E.4 Group Tear-Out Capacity

The adjusted tear-out capacity of a group of "n" rows of fasteners can be estimated as:

$$Z_{GT}' = \frac{Z_{RT-1}'}{2} + \frac{Z_{RT-n}'}{2} + F_t' A_{group-net} \qquad \text{(E.4-1)}$$

where:

Z_{GT}' = adjusted group tear-out capacity

Z_{RT-1}' = adjusted row tear-out capacity of row 1 of fasteners bounding the critical group area

Z_{RT-n}' = adjusted row tear-out capacity of row n of fasteners bounding the critical group area

$A_{group-net}$ = critical group net section area between row 1 and row n

E.4.1 For groups of fasteners with non-uniform spacing between rows of fasteners various definitions of critical group area should be checked for group tear-out in combination with row tear-out to determine the adjusted capacity of the critical section.

E.5 Effects of Fastener Placement

E.5.1 Modification of fastener placement within a fastener group can be used to increase row tear-out and group tear-out capacity limited by local stresses around the fastener group. Increased spacing between fasteners in a row is one way to increase row tear-out capacity. Increased spacing between rows of fasteners is one way to increase group tear-out capacity.

E.5.2 Section 12.5.1.3 limits the spacing between outer rows of fasteners paralleling the member on a single splice plate to 5 inches. This requirement is imposed to limit local stresses resulting from shrinkage of wood members. Where special detailing is used to address shrinkage, such as the use of slotted holes, the 5-inch limit can be adjusted.

E.6 Sample Solution of Staggered Bolts

Calculate the net section area tension, row tear-out, and group tear-out ASD adjusted design capacities for the double-shear bolted connection in Figure E1.

Main Member:
Combination 2 Douglas fir 3-1/8 x 12 glued laminated timber member. See *NDS Supplement* Table 5B – Members stressed primarily in axial tension or compression for reference design values. Adjustment factors C_D, C_T, and C_M are assumed to equal 1.0 and C_{vr} = 0.72 (see NDS 5.3.10) is used in this example for calculation of adjusted design values.

F_t' = 1250 psi

F_v' = 265 psi (C_{vr}) = 265 (0.72) = 191 psi

Main member thickness, t_m: 3.125 in.
Main member width, w: 12 in.

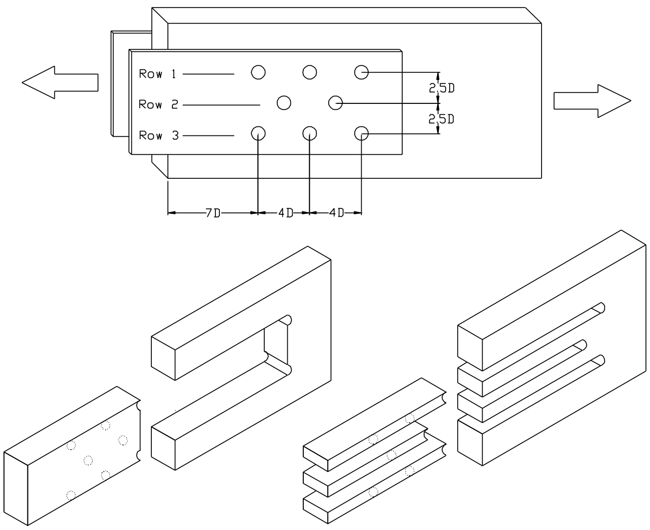
Side Member:
A36 steel plates on each side
Side plate thickness, t_s: 0.25 in.

Connection Details:
Bolt diameter, D: 1 inch
Bolt hole diameter, D_h: 1.0625 in.
Adjusted ASD bolt design value, $Z_{||}'$: 4380 lbs (see Table 12I. For this trial design, the group action factor, C_g, is taken as 1.0).
Spacing between rows: s_{row} = 2.5D

Adjusted ASD Connection Capacity, $nZ_{||}'$:

$nZ_{||}'$ = (8 bolts)(4,380 lbs) = 35,040 lbs

Figure E1 Staggered Rows of Bolts



Adjusted ASD Net Section Area Tension Capacity, Z_{NT}':

$$Z_{NT}' = F_t' t\left[w - n_{row} D_h\right]$$

Z_{NT}' = (1,250 psi)(3.125")[12" – 3(1.0625")]

= 34,424 lbs

Adjusted ASD Row Tear-Out Capacity, Z_{RT}':

$$Z_{RTi}' = n_i F_v' t s_{critical}$$

$Z_{RT\text{-}1}'$ = 3(191 psi)(3.125")(4") = 7,163 lbs

$Z_{RT\text{-}2}'$ = 2(191 psi)(3.125")(4") = 4,775 lbs

$Z_{RT\text{-}3}'$ = 3(191 psi)(3.125")(4") = 7,163 lbs

$$Z_{RT}' = \sum_{i=1}^{n_{row}} Z_{RTi}'$$

= 7,163 + 4,775 + 7,163 = 19,101 lbs

Adjusted ASD Group Tear-Out Capacity, Z_{GT}':

$$Z_{GT}' = \frac{Z_{RT-1}'}{2} + \frac{Z_{RT-3}'}{2} + F_t' t\left[\left(n_{row} - 1\right)\left(s_{row} - D_h\right)\right]$$

Z_{GT}' = (7,163 lbs)/2 + (7,163 lbs)/2 +

(1,250 psi) (3.125")[(3 – 1)(2.5" – 1.0625")]

= 18,393 lbs

In this sample calculation, the adjusted ASD connection capacity is limited to 18,393 pounds by group tear-out, Z_{GT}'.

E.7 Sample Solution of Row of Bolts

Calculate the net section area tension and row tear-out adjusted ASD design capacities for the single-shear single-row bolted connection represented in Figure E2.

Main and Side Members:
#2 grade Hem-Fir lumber. See *NDS Supplement* Table 4A – Visually Graded Dimension Lumber for reference design values. Adjustment factors C_D, C_T, C_M, and C_i are assumed to equal 1.0 in this example for calculation of adjusted design values.

F_t' = 525 psi (C_F) = 525(1.5) = 788 psi

F_v' = 150 psi

Connection Details:
Bolt diameter, D: 1/2 in.
Bolt hole diameter, D_h: 0.5625 in.
Adjusted ASD bolt design value, $Z_{\|}'$: 550 lbs (See NDS Table 12A for 3-1/2" main member thickness and 1-1/2" side member thickness. For this trial design, the group action factor, C_g, is taken as 1.0).

Adjusted ASD Connection Capacity, $nZ_{\|}'$:

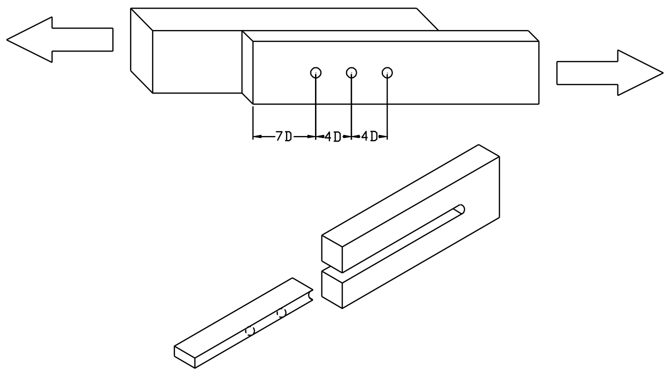
$nZ_{||}'$ = (3 bolts)(550 lbs) = 1,650 lbs

For side member, adjusted ASD Net Section Area Tension Capacity, Z_{NT}':

$$Z_{NT}' = F_t' t\left[w - n_{row} D_h\right]$$

Z_{NT}' = (788 psi)(1.5")[3.5" – 1(0.5625")] = 3,470 lbs

Figure E2 Single Row of Bolts



For side member, adjusted ASD Row Tear-Out Capacity, Z_{RT}':

$$Z_{RTi}' = n_i F_v' t s_{critical}$$

Z_{RT1}' = 3(150 psi)(1.5")(2") = 1,350 lbs

In this sample calculation, the adjusted ASD connection capacity is limited to 1,350 pounds by row tear-out, Z_{RT}'.

E.8 Sample Solution of Row of Split Rings

Calculate the net section area tension and row tear-out adjusted ASD design capacities for the single-shear single-row split ring connection represented in Figure E3.

Main and Side Members:
#2 grade Southern Pine 2x4 lumber. See *NDS Supplement* Table 4B – Visually Graded Southern Pine Dimension Lumber for reference design values. Adjustment factors C_D, C_T, C_M, and C_i are assumed to equal 1.0 in this example for calculation of adjusted design values.

$F_t' = 675$ psi

$F_v' = 175$ psi

Main member thickness, t_m: 1.5 in.
Side member thickness, t_s: 1.5 in.
Main and side member width, w: 3.5 in.

Connection Details:
Split ring diameter, D: 2.5 in. (see Appendix K for connector dimensions)
Adjusted ASD split ring design value, P′: 2,730 lbs (see Table 13.2A. For this trial design, the group action factor, C_g, is taken as 1.0).

Adjusted ASD Connection Capacity, nP′:

nP′ = (2 split rings)(2,730 lbs) = 5,460 lbs

Adjusted ASD Net Section Area Tension Capacity, Z_{NT}':

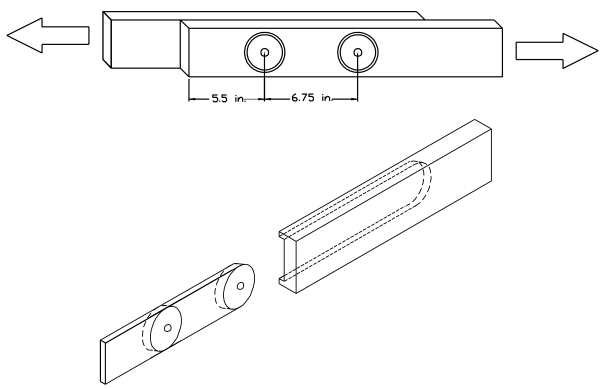
$$Z_{NT}' = F_t' A_{net}$$

$Z_{NT}' = F_t'\ [A_{2x4} - A_{bolt\text{-}hole} - A_{split\ ring\ projected\ area}]$

$Z_{NT}' = (675\text{ psi})[5.25\text{ in.}^2 - 1.5"\ (0.5625") - 1.1\text{ in.}^2]$

$= 2{,}232$ lbs

Figure E3 Single Row of Split Ring Connectors



Adjusted ASD Row Tear-Out Capacity, Z_{RT}':

$$Z_{RTi}' = n_i \frac{F_v' A_{critical}}{2}$$

Z_{RT1}' = [(2 connectors)(175 psi)/2](21.735 in.²)

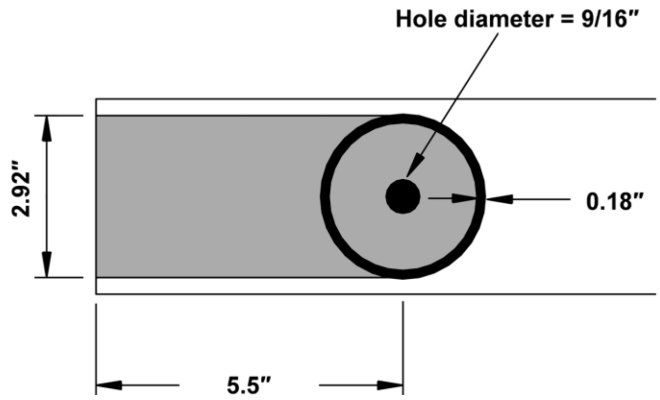
= 3,804 lbs

where:

$A_{critical} = 21.735$ in.² (See Figures E4 and E5)

In this sample calculation, the adjusted ASD connection capacity is limited to 2,232 pounds by net section area tension capacity, Z_{NT}'.

Figure E4 $A_{critical}$ for Split Ring Connection (based on distance from end of member)



$A_{edge\ plane}$ = (2 shear lines) (groove depth)($s_{critical}$)

= (2 shear lines) (0.375")(5.5") = 4.125 in.²

$A_{bottom\ plane\ net} = (A_{bottom\ plane}) - (A_{split\ ring\ groove}) - (A_{bolt\ hole})$

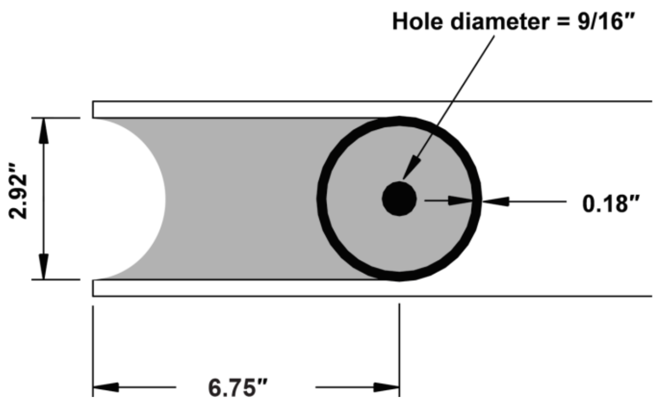
$= [(5.5")(2.92") + (\pi)(2.92")^2/8] - (\pi/4)[(2.92")^2 - (2.92" - 0.18" - 0.18")^2] - (\pi/4)(0.5625")^2$

= 17.61 in.²

$A_{criticial} = A_{edge\ plane} + A_{bottom\ plane\ net}$

= 21.735 in.²

Figure E5 **$A_{critical}$ for Split Ring Connection (based on distance between first and second split ring)**



$A_{edge\ plane}$ = (2 shear lines) (groove depth)($s_{critical}$)

= (2 shear lines) (0.375")(6.75") = 5.063 in.^{2}

$A_{bottom\ plane\ net}$ = ($A_{bottom\ plane}$) – ($A_{split\ ring\ groove}$) – ($A_{bolt\ hole}$)

= (6.75")(2.92") – (π/4)[(2.92")^2–(2.92" – 0.18" – 0.18")^2] – (π/4)(0.5625")^2

= 17.91 in.^{2}

$A_{criticial}$ = $A_{edge\ plane}$ + $A_{bottom\ plane\ net}$

= 5.063 + 17.91 in.^{2} = 22.973 in.^{2}

Therefore $A_{critical}$ is governed by the case shown in Figure E4 and is equal to 21.735 in.^{2}

Appendix F (Non-mandatory) Design for Creep and Critical Deflection Applications

F.1 Creep

F.1.1 Reference modulus of elasticity design values, E, in this Specification are intended for the calculation of immediate deformation under load. Under sustained loading, wood members exhibit additional time dependent deformation (creep) which usually develops at a slow but persistent rate over long periods of time. Creep rates are greater for members drying under load or exposed to varying temperature and relative humidity conditions than for members in a stable environment and at constant moisture content.

F.1.2 In certain bending applications, it may be necessary to limit deflection under long-term loading to specified levels. This can be done by applying an increase factor to the deflection due to long-term load. Total deflection is thus calculated as the immediate deflection due to the long-term component of the design load times the appropriate increase factor, plus the deflection due to the short-term or normal component of the design load.

F.2 Variation in Modulus of Elasticity

F.2.1 The reference modulus of elasticity design values, E, listed in Tables 4A, 4B, 4C, 4D, 4E, 4F, 5A, 5B, 5C, and 5D (published in the Supplement to this Specification) are average values and individual pieces having values both higher and lower than the averages will occur in all grades. The use of average modulus of elasticity values is customary practice for the design of normal wood structural members and assemblies. Field experience and tests have demonstrated that average values provide an adequate measure of the immediate deflection or deformation of these wood elements.

F.2.2 In certain applications where deflection may be critical, such as may occur in closely engineered, innovative structural components or systems, use of a reduced modulus of elasticity value may be deemed appropriate by the designer. The coefficient of variation in Table F1 shall be permitted to be used as a basis for modifying reference modulus of elasticity values listed in Tables 4A, 4B, 4C, 4D, 4E, 4F, 5A, 5B, 5C, and 5D to meet particular end use conditions.

F.2.3 Reducing reference average modulus of elasticity design values in this Specification by the product of the average value and 1.0 and 1.65 times the applicable coefficients of variation in Table F1 gives estimates of the level of modulus of elasticity exceeded by 84% and 95%, respectively, of the individual pieces, as specified in the following formulas:

$$E_{0.16} = E\left(1 - 1.0\ COV_E\right) \qquad \text{(F-1)}$$

$$E_{0.05} = E\left(1 - 1.645\ COV_E\right) \qquad \text{(F-2)}$$

Table F1 Coefficients of Variation in Modulus of Elasticity (COV_E) for Lumber and Structural Glued Laminated Timber

	COV_E
Visually graded sawn lumber (Tables 4A, 4B, 4D, 4E, and 4F)	0.25
Machine Evaluated Lumber (MEL) (Table 4C)	0.15
Machine Stress Rated (MSR) lumber (Table 4C)	0.11
Structural glued laminated timber (Tables 5A, 5B, 5C, and 5D)	0.10

F.3 Shear Deflection

F.3.1 Reference modulus of elasticity design values, E, listed in Tables 4A, 4B, 4C, 4D, 4E, 4F, 5A, 5B, 5C, and 5D are apparent modulus of elasticity values and include a shear deflection component. For sawn lumber, the ratio of shear-free E to reference E is 1.03. For structural glued laminated timber, the ratio of shear-free E to reference E is 1.05.

F.3.2 In certain applications use of an adjusted modulus of elasticity to more accurately account for the shear component of the total deflection may be deemed appropriate by the designer. Standard methods for adjusting modulus of elasticity to other load and span-depth conditions are available (see Reference 54). When reference modulus of elasticity values have not been adjusted to include the effects of shear deformation, such as for prefabricated wood I-joists, consideration for the shear component of the total deflection is required.

F.3.3 The shear component of the total deflection of a beam is a function of beam geometry, modulus of elasticity, shear modulus, applied load and support conditions. The ratio of shear-free E to apparent E is

1.03 for the condition of a simply supported rectangular beam with uniform load, a span to depth ratio of 21:1, and elastic modulus to shear modulus ratio of 16:1. The ratio of shear-free E to apparent E is 1.05 for a similar beam with a span to depth ratio of 17:1. See Reference 53 for information concerning calculation of beam deflection for other span-depth and load conditions.

Appendix G (Non-mandatory) Effective Column Length

G.1

The effective column length of a compression member is the distance between two points along its length at which the member is assumed to buckle in the shape of a sine wave.

G.2

The effective column length is dependent on the values of end fixity and lateral translation (deflection) associated with the ends of columns and points of lateral support between the ends of column. It is recommended that the effective length of columns be determined in accordance with good engineering practice. Lower values of effective length will be associated with more end fixity and less lateral translation while higher values will be associated with less end fixity and more lateral translation.

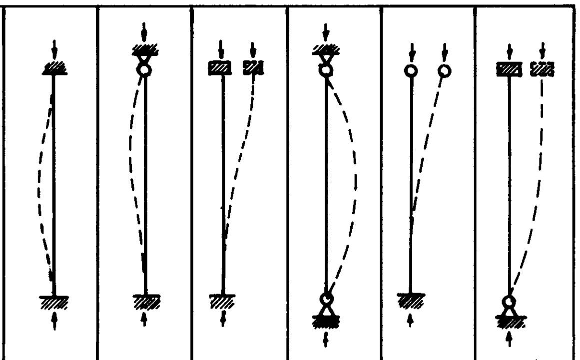
G.3

In lieu of calculating the effective column length from available engineering experience and methodology, the buckling length coefficients, K_e, given in Table G1 shall be permitted to be multiplied by the actual column length, ℓ, or by the length of column between lateral supports to calculate the effective column length, ℓ_e.

G.4

Where the bending stiffness of the frame itself provides support against buckling, the buckling length coefficient, K_e, for an unbraced length of column, ℓ, is dependent upon the amount of bending stiffness provided by the other in-plane members entering the connection at each end of the unbraced segment. If the combined stiffness from these members is sufficiently small relative to that of the unbraced column segments, K_e could exceed the values given in Table G1.

Table G1 Buckling Length Coefficients, K_e

Buckling modes						
Theoretical K_e value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design K_e when ideal conditions approximated	0.65	0.80	1.2	1.0	2.10	2.4
End condition code	Rotation fixed, translation fixed Rotation free, translation fixed Rotation fixed, translation free Rotation free, translation free					

Appendix H (Non-mandatory) Lateral Stability of Columns

H.1

Solid wood columns can be classified into three length classes, characterized by mode of failure at ultimate load. For short, rectangular columns with a small ratio of length to least cross-sectional dimension, ℓ_e/d, failure is by crushing. When there is an intermediate ℓ_e/d ratio, failure is generally a combination of crushing and buckling. At large ℓ_e/d ratios, long wood columns behave essentially as Euler columns and fail by lateral deflection or buckling. Design of these three length classes are represented by the single column Equation H-1.

H.2

For solid columns of rectangular cross section where the slenderness ratio, ℓ_e/d, does not exceed 50, adjusted compression design values parallel to grain are obtained by the equation:

$$F_c' = F_c^* \left[\frac{1+\left(F_{cE}/F_c^*\right)}{2c} - \sqrt{\left[\frac{1+\left(F_{cE}/F_c^*\right)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}} \right] \quad \text{(H-1)}$$

where:

$$F_{cE} = \frac{0.822\ E_{min}'}{\left(\ell_e/d\right)^2} \quad \text{(H-2)}$$

F_c^* = reference compression design value parallel to grain multiplied by all applicable adjustment factors except C_P (see 2.3)

c = 0.8 for sawn lumber

c = 0.85 for round timber poles and piles

c = 0.9 for structural glued laminated timber, cross-laminated timber, or structural composite lumber

Equation H-2 is derived from the standard Euler equation, with radius of gyration, r, converted to the more convenient least cross-sectional dimension, d, of a rectangular column.

H.3

The equation for adjusted compression design value, F_c', in this Specification is for columns having rectangular cross sections. It may be used for other column shapes by substituting $r\sqrt{12}$ for d in the equations, where r is the applicable radius of gyration of the column cross section.

H.4

The 0.822 factor in Equation H-2 represents the Euler buckling coefficient for rectangular columns calculated as $\pi^2/12$. Modulus of elasticity for beam and column stability, E_{min}, in Equation H-2 represents an approximate 5% lower exclusion value on pure bending modulus of elasticity, plus a 1.66 factor of safety (see Appendix D.4).

H.5

Adjusted design values based on Equations H-1 and H-2 are customarily used for most sawn lumber column designs. Where unusual hazard exists, a larger reduction factor may be appropriate. Alternatively, in less critical end use, the designer may elect to use a smaller factor of safety.

H.6

For products with less E variability than visually graded sawn lumber, higher critical buckling design values may be calculated. For a product having a lower coefficient of variation (COV_E), use of Equation H-2 will provide a 1.66 factor of safety at the 5% lower exclusion value.

Appendix I (Non-mandatory) Yield Limit Equations for Connections

I.1 Yield Modes

The yield limit equations specified in 12.3.1 for dowel-type fasteners such as bolts, lag screws, wood screws, nails, and spikes represent four primary connection yield modes (see Figure I1). Modes I_m and I_s represent bearing-dominated yield of the wood fibers in contact with the fastener in either the main or side member(s), respectively. Mode II represents pivoting of the fastener at the shear plane of a single shear connection with localized crushing of wood fibers near the faces of the wood member(s). Modes III_m and III_s represent fastener yield in bending at one plastic hinge point per shear plane, and bearing-dominated yield of wood fibers in contact with the fastener in either the main or side member(s), respectively. Mode IV represents fastener yield in bending at two plastic hinge points per shear plane, with limited localized crushing of wood fibers near the shear plane(s).

I.2 Dowel Bearing Strength for Steel Members

Dowel bearing strength, F_e, for steel members shall be based on accepted steel design practices (see References 39, 40 and 41). Design values in Tables 12B, 12D, 12G, 12I, 12K, 12M, 12P, and 12T are for 1/4" ASTM A 36 steel plate or 3 gage and thinner ASTM A 653, Grade 33 steel plate with dowel bearing strength proportional to ultimate tensile strength. Bearing strengths used to calculate connection yield load represent nominal bearing strengths of 2.4 F_u and 2.2 F_u, respectively (based on design provisions in References 39, 40, and 41 for bearing strength of steel members at connections). To allow proper application of the load duration factor for these connections, the bearing strengths have been divided by 1.6.

I.3 Dowel Bearing Strength for Wood Members

Dowel bearing strength, F_e, for wood members may be determined in accordance with ASTM D 5764.

I.4 Fastener Bending Yield Strength, F_{yb}

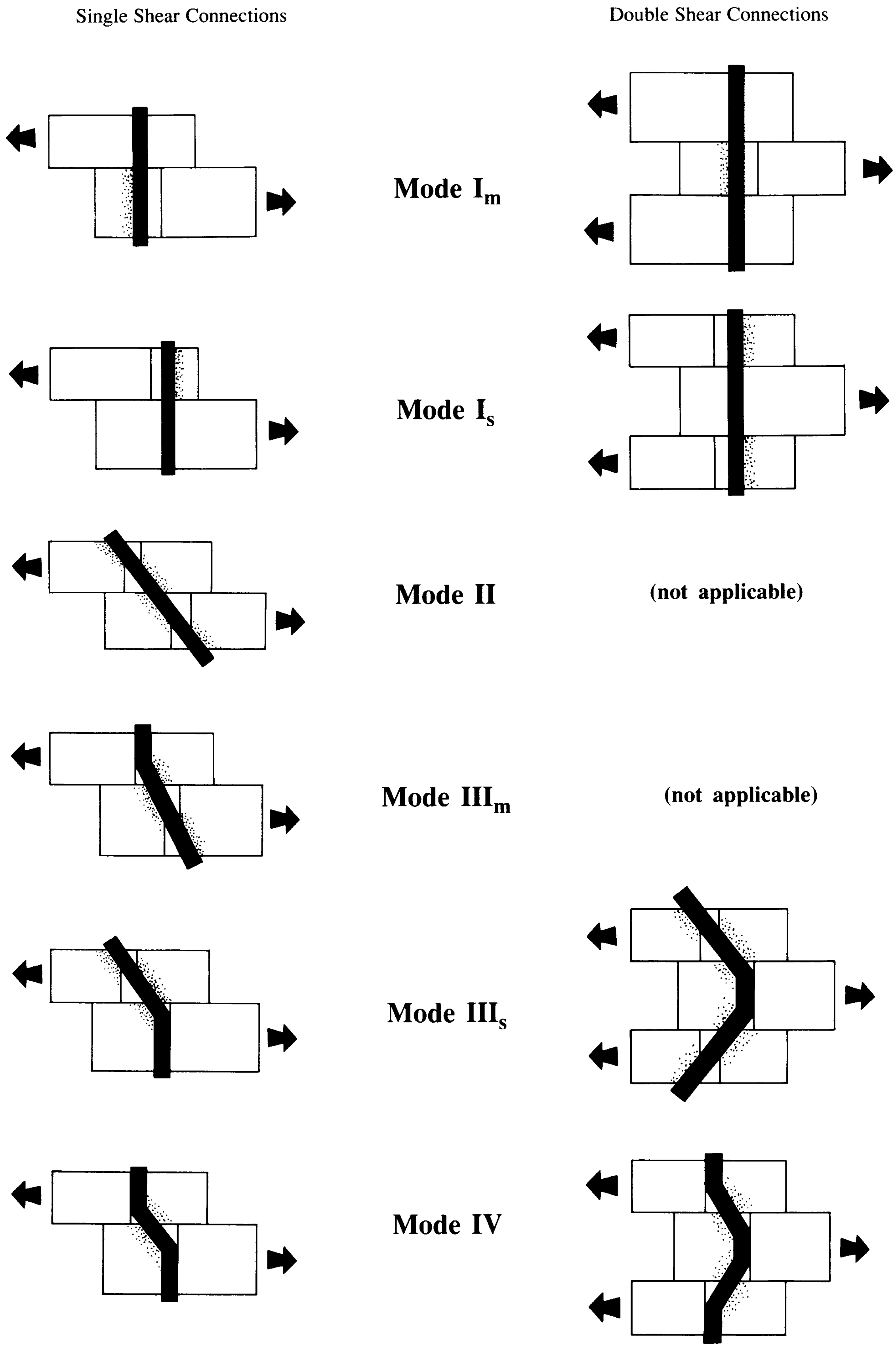
In the absence of published standards which specify fastener strength properties, the designer should contact fastener manufacturers to determine fastener bending yield strength for connection design. ASTM F 1575 provides a standard method for testing bending yield strength of nails.

Fastener bending yield strength (F_{yb}) shall be determined by the 5% diameter (0.05D) offset method of analyzing load-displacement curves developed from fastener bending tests. However, for short, large diameter fasteners for which direct bending tests are impractical, test data from tension tests such as those specified in ASTM F 606 shall be evaluated to estimate F_{yb}.

Research indicates that F_{yb} for bolts is approximately equivalent to the average of bolt tensile yield strength and bolt tensile ultimate strength, $F_{yb} = F_y/2 + F_u/2$. Based on this approximation, 48,000 psi $\leq F_{yb} \leq$ 140,000 psi for various grades of SAE J429 bolts. Thus, the aforementioned research indicates that F_{yb} = 45,000 psi is reasonable for many commonly available bolts. Tests of limited samples of lag screws indicate that F_{yb} = 45,000 psi is also reasonable for many commonly available lag screws with $D \geq 3/8"$.

Tests of a limited sample of box nails and common wire nails from twelve U.S. nail manufacturers indicate that F_{yb} increases with decreasing nail diameter, and may exceed 100,000 psi for very small diameter nails. These tests indicate that the F_{yb} values used in Tables 12N through 12R are reasonable for many commonly available box nails and small diameter common wire nails ($D < 0.2"$). Design values for large diameter common wire nails ($D > 0.2"$) are based on extrapolated estimates of F_{yb} from the aforementioned limited study. For hardened-steel nails, F_{yb} is assumed to be approximately 30% higher than for the same diameter common wire nails. Design values in Tables 12J through 12M for wood screws and small diameter lag screws ($D < 3/8"$) are based on estimates of F_{yb} for common wire nails of the same diameter. Table I1 provides values of F_{yb} based on fastener type and diameter.

Figure I1 (Non-mandatory) Connection Yield Modes



I.5 Threaded Fasteners

The reduced moment resistance in the threaded portion of dowel-type fasteners can be accounted for by use of root diameter, D_r, in calculation of reference lateral design values. Use of diameter, D, is permitted when the threaded portion of the fastener is sufficiently far away from the connection shear plane(s). For example, diameter, D, may be used when the length of thread bearing in the main member of a two member connection does not exceed 1/4 of the total bearing length in the main member (member holding the threads). For a connection with three or more members, diameter, D, may be used when the length of thread bearing in the outermost member does not exceed 1/4 of the total bearing length in the outermost member (member holding the threads).

Reference lateral design values for reduced body diameter lag screw and rolled thread wood screw connections are based on root diameter, D_r to account for the reduced diameter of these fasteners. These values may also be applicable for full-body diameter lag screws and cut thread wood screws since the length of threads for these fasteners is generally not known and/or the thread bearing length based on typical dimensions exceeds 1/4 the total bearing length in the member holding the threads. For bolted connections, reference tabulated lateral design values are based on diameter, D.

One alternate method of accounting for the moment and bearing resistance of the threaded portion of the fastener and moment acting along the length of the fastener is provided in AWC's *Technical Report 12 - General Dowel Equations for Calculating Lateral Connection Values* (see Reference 51). A general set of equations permits use of different fastener diameters for bearing resistance and moment resistance in each member.

Table I1 Fastener Bending Yield Strengths, F_{yb}

Fastener Type	F_{yb} (psi)
Bolt, lag screw (with D ≥ 3/8"), drift pin (SAE J429 Grade 1 - F_y = 36,000 psi and F_u = 60,000 psi)	45,000
Common, box, or sinker nail, spike, lag screw, wood screw (low to medium carbon steel)	
0.099" ≤ D ≤ 0.142"	100,000
0.142" < D ≤ 0.177"	90,000
0.177" < D ≤ 0.236"	80,000
0.236" < D ≤ 0.273"	70,000
0.273" < D ≤ 0.344"	60,000
0.344" < D ≤ 0.375"	45,000
Hardened steel nail (medium carbon steel) including post-frame ring shank nails	
0.120" ≤ D ≤ 0.142"	130,000
0.142" < D ≤ 0.192"	115,000
0.192" < D ≤ 0.207"	100,000

Appendix J (Non-mandatory) Solution of Hankinson Formula

J.1

When members are loaded in bearing at an angle to grain between 0° and 90°, or when split ring or shear plate connectors, bolts, or lag screws are loaded at an angle to grain between 0° and 90°, design values at an angle to grain shall be determined using the Hankinson formula.

J.2

The Hankinson formula is for the condition where the loaded surface is perpendicular to the direction of the applied load.

J.3

When the resultant force is not perpendicular to the surface under consideration, the angle θ is the angle between the direction of grain and the direction of the force component which is perpendicular to the surface.

J.4

The bearing surface for a split ring or shear plate connector, bolt or lag screw is assumed perpendicular to the applied lateral load.

J.5

The bearing strength of wood depends upon the direction of grain with respect to the direction of the applied load. Wood is stronger in compression parallel to grain than in compression perpendicular to grain. The variation in strength at various angles to grain between 0° and 90° shall be determined by the Hankinson formula as follows:

$$F_\theta' = \frac{F_c^* F_{c\perp}'}{F_c^* \sin^2\theta + F_{c\perp}' \cos^2\theta} \qquad \text{(J-1)}$$

where:

F_c^* = adjusted compression design value parallel to grain multiplied by all applicable adjustment factors except the column stability factor

$F_{c\perp}'$ = adjusted compression design value perpendicular to grain

F_θ' = adjusted bearing design value at an angle to grain

θ = angle between direction of load and direction of grain (longitudinal axis of member)

When determining dowel bearing design values at an angle to grain for bolt or lag screw connections, the Hankinson formula takes the following form:

$$F_{e\theta} = \frac{F_{e\parallel} F_{e\perp}}{F_{e\parallel} \sin^2\theta + F_{e\perp} \cos^2\theta} \qquad \text{(J-2)}$$

where:

$F_{e\parallel}$ = dowel bearing strength parallel to grain

$F_{e\perp}$ = dowel bearing strength perpendicular to grain

$F_{e\theta}$ = dowel bearing strength at an angle to grain

When determining adjusted design values for bolt or lag screw wood-to-metal connections or wood-to-wood connections with the main or side member(s) loaded parallel to grain, the following form of the Hankinson formula provides an alternate solution:

$$Z_\theta' = \frac{Z_\parallel' Z_\perp'}{Z_\parallel' \sin^2\theta + Z_\perp' \cos^2\theta} \qquad \text{(J-3)}$$

For wood-to-wood connections with side member(s) loaded parallel to grain,

$Z_\parallel'$ = adjusted lateral design value for a single bolt or lag screw connection with the main and side wood members loaded parallel to grain, $Z_\parallel$

$Z_\perp'$ = adjusted lateral design value for a single bolt or lag screw connection with the side member(s) loaded parallel to grain and main member loaded perpendicular to grain, $Z_{m\perp}$

For wood-to-wood connections with the main member loaded parallel to grain,

$Z_{\parallel}'$ = adjusted lateral design value for a single bolt or lag screw connection with the main and side wood members loaded parallel to grain, $Z_{\parallel}$

$Z_{\perp}'$ = adjusted lateral design value for a single bolt or lag screw connection with the main member loaded parallel to grain and side member(s) loaded perpendicular to grain, $Z_{s\perp}$

For wood-to-metal connections,

$Z_{\parallel}'$ = adjusted lateral design value for a single bolt or lag screw connection with the wood member loaded parallel to grain, $Z_{\parallel}$

$Z_{\perp}'$ = adjusted lateral design value for a single bolt or lag screw connection with the wood member loaded perpendicular to grain, $Z_{\perp}$

When determining adjusted design values for split ring or shear plate connectors or timber rivets, the Hankinson formula takes the following form:

$$N' = \frac{P'Q'}{P' \sin^2 \theta + Q' \cos^2 \theta} \tag{J-4}$$

where:

P' = adjusted lateral design value parallel to grain for a single split ring connector unit or shear plate connector unit

Q' = adjusted lateral design value perpendicular to grain for a single split ring connector unit or shear plate connector unit

N' = adjusted lateral design value at an angle to grain for a single split ring connector unit or shear plate connector unit

The nomographs presented in Figure J1 provide a graphical solution of the Hankinson formula.

Figure J1 Solution of Hankinson Formula

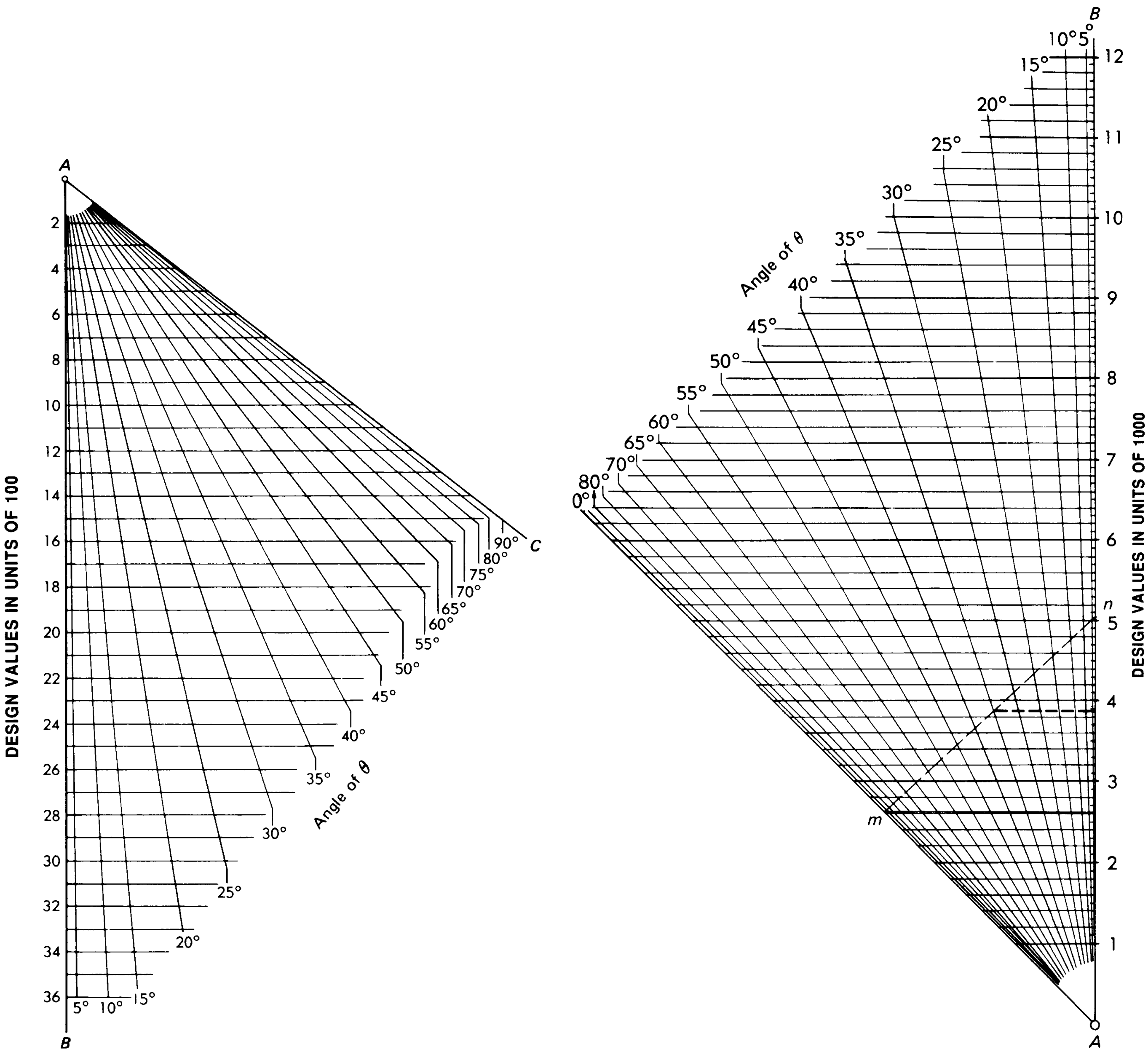
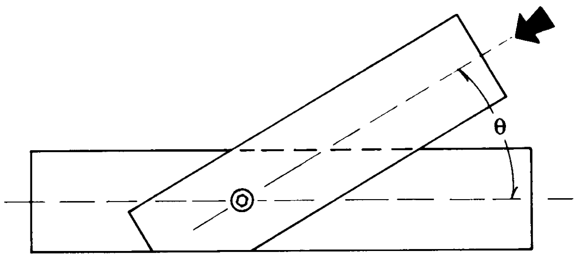


Figure J2 Connection Loaded at an Angle to Grain



Sample Solution for Split Ring or Shear Plate Connection:

Assume that P' = 5,030 lbs, Q' = 2,620 lbs, and θ = 35° in Figure J2. On line A-B in Figure J1, locate 5,030 lbs at point n. On the same line A-B, locate 2,620 lbs and project to point m on line A-C. Where line m-n intersects the radial line for 35°, project to line A-B and read the ASD adjusted design value, N' = 3,870 lbs.

Appendix K (Non-mandatory) Typical Dimensions for Split Ring and Shear Plate Connectors

SPLIT RINGS[1]	2-1/2"	4"
Split Ring		
Inside diameter at center when closed	2.500"	4.000"
Thickness of metal at center	0.163"	0.193"
Depth of metal (width of ring)	0.750"	1.000"
Groove		
Inside diameter	2.56"	4.08"
Width	0.18"	0.21"
Depth	0.375"	0.50"
Bolt hole diameter in timber members	9/16"	13/16"
Washers, standard		
Round, cast or malleable iron, diameter	2-1/8"	3"
Round, wrought iron (minimum)		
Diameter	1-3/8"	2"
Thickness	3/32"	5/32"
Square plate		
Length of side	2"	3"
Thickness	1/8"	3/16"
Projected area: portion of one split ring within member	1.10 in.^{2}	2.24 in.^{2}

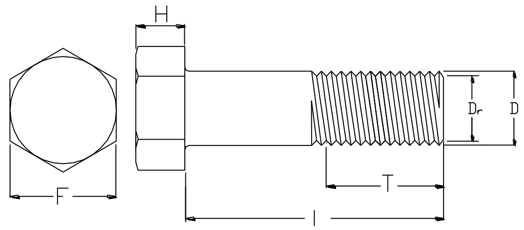
1. Courtesy of Cleveland Steel Specialty Co.

SHEAR PLATES	2-5/8"	2-5/8"	4"	4"
Shear plate[1]				
Material	Pressed steel	Malleable cast iron	Malleable cast iron	Malleable cast iron
Plate diameter	2.62"	2.62"	4.02"	4.02"
Bolt hole diameter	0.81"	0.81"	0.81"	0.93"
Plate thickness	0.172"	0.172"	0.20"	0.20"
Plate depth	0.42"	0.42"	0.62"	0.62"
Bolt hole diameter in timber members and metal side plates[2]	13/16"	13/16"	13/16"	15/16"
Washers, standard				
Round, cast or malleable iron, diameter	3"	3"	3"	3-1/2"
Round, wrought iron (minimum)				
Diameter	2"	2"	2"	2-1/4"
Thickness	5/32"	5/32"	5/32"	11/64"
Square plate				
Length of side	3"	3"	3"	3"
Thickness	1/4"	1/4"	1/4"	1/4"
Projected area: portion of one shear plate within member	1.18 in.^{2}	1.00 in.^{2}	2.58 in.^{2}	2.58 in.^{2}

1. ASTM D 5933.
2. Steel straps or shapes used as metal side plates shall be designed in accordance with accepted metal practices (see 11.2.3).

Appendix L (Non-mandatory) Typical Dimensions for Dowel-Type Fasteners and Washers[1]

Table L1 Standard Hex Bolts[1]



Full-Body
Diameter

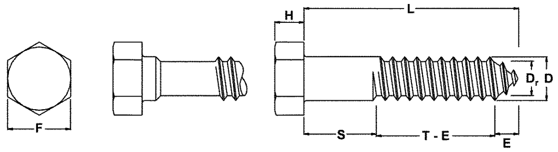
D = diameter, in.
D_r = root diameter, in.
T = thread length, in.
L = bolt length, in.
F = width of head across flats, in.
H = height of head, in.

		Diameter, D							
		1/4	**5/16**	**3/8**	**1/2**	**5/8**	**3/4**	**7/8**	**1**
D_r		0.189	0.245	0.298	0.406	0.514	0.627	0.739	0.847
F		7/16	1/2	9/16	3/4	15/16	1-1/8	1-5/16	1-1/2
H		11/64	7/32	1/4	11/32	27/64	1/2	37/64	43/64
T	L ≤ 6	3/4	7/8	1	1-1/4	1-1/2	1-3/4	2	2-1/4
	L > 6	1	1-1/8	1-1/4	1-1/2	1-3/4	2	2-1/4	2-1/2

1. Tolerances are specified in ANSI/ASME B18.2.1. Full-body diameter bolt is shown. Root diameter based on UNC thread series (see ANSI/ASME B1.1).

Table L2 Standard Hex Lag Screws[1]

D = diameter, in.
D_r = root diameter, in.
S = unthreaded body length, in.
T = minimum thread length[2], in.



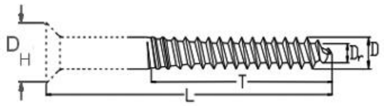
Reduced Body Diameter   Full-Body Diameter

E = length of tapered tip, in.
L = lag screw length, in.
N = number of threads/inch
F = width of head across flats, in.
H = height of head, in.

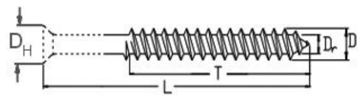
Length, L		Diameter, D										
		1/4	5/16	3/8	7/16	1/2	5/8	3/4	7/8	1	1-1/8	1-1/4
	D_r	0.173	0.227	0.265	0.328	0.371	0.471	0.579	0.683	0.780	0.887	1.012
	E	5/32	3/16	7/32	9/32	5/16	13/32	1/2	19/32	11/16	25/32	7/8
	H	11/64	7/32	1/4	19/64	11/32	27/64	1/2	37/64	43/64	3/4	27/32
	F	7/16	1/2	9/16	5/8	3/4	15/16	1-1/8	1-5/16	1-1/2	1-11/16	1-7/8
	N	10	9	7	7	6	5	4-1/2	4	3-1/2	3-1/4	3-1/4
1	S	1/4	1/4	1/4	1/4	1/4						
	T	3/4	3/4	3/4	3/4	3/4						
	T-E	19/32	9/16	17/32	15/32	7/16						
1-1/2	S	1/4	1/4	1/4	1/4	1/4						
	T	1-1/4	1-1/4	1-1/4	1-1/4	1-1/4						
	T-E	1-3/32	1-1/16	1-1/32	31/32	15/16						
2	S	1/2	1/2	1/2	1/2	1/2	1/2					
	T	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2					
	T-E	1-11/32	1-5/16	1-9/32	1-7/32	1-3/16	1-3/32					
2-1/2	S	3/4	3/4	3/4	3/4	3/4	3/4					
	T	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4	1-3/4					
	T-E	1-19/32	1-9/16	1-17/32	1-15/32	1-7/16	1-11/32					
3	S	1	1	1	1	1	1	1	1	1		
	T	2	2	2	2	2	2	2	2	2		
	T-E	1-27/32	1-13/16	1-25/32	1-23/32	1-11/16	1-19/32	1-1/2	1-13/32	1-5/16		
4	S	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2	1-1/2
	T	2-1/2	2-1/2	2-1/2	2-1/2	2-1/2	2-1/2	2-1/2	2-1/2	2-1/2	2-1/2	2-1/2
	T-E	2-11/32	2-5/16	2-9/32	2-7/32	2-3/16	2-3/32	2	1-29/32	1-13/16	1-23/32	1-5/8
5	S	2	2	2	2	2	2	2	2	2	2	2
	T	3	3	3	3	3	3	3	3	3	3	3
	T-E	2-27/32	2-13/16	2-25/32	2-23/32	2-11/16	2-19/32	2-1/2	2-13/32	2-5/16	2-7/32	2-1/8
6	S	2-1/2	2-1/2	2-1/2	2-1/2	2-1/2	2-1/2	2-1/2	2-1/2	2-1/2	2-1/2	2-1/2
	T	3-1/2	3-1/2	3-1/2	3-1/2	3-1/2	3-1/2	3-1/2	3-1/2	3-1/2	3-1/2	3-1/2
	T-E	3-11/32	3-5/16	3-9/32	3-7/32	3-3/16	3-3/32	3	2-29/32	2-13/16	2-23/32	2-5/8
7	S	3	3	3	3	3	3	3	3	3	3	3
	T	4	4	4	4	4	4	4	4	4	4	4
	T-E	3-27/32	3-13/16	3-25/32	3-23/32	3-11/16	3-19/32	3-1/2	3-13/32	3-5/16	3-7/32	3-1/8
8	S	3-1/2	3-1/2	3-1/2	3-1/2	3-1/2	3-1/2	3-1/2	3-1/2	3-1/2	3-1/2	3-1/2
	T	4-1/2	4-1/2	4-1/2	4-1/2	4-1/2	4-1/2	4-1/2	4-1/2	4-1/2	4-1/2	4-1/2
	T-E	4-11/32	4-5/16	4-9/32	4-7/32	4-3/16	4-3/32	4	3-29/32	3-13/16	3-23/32	3-5/8
9	S	4	4	4	4	4	4	4	4	4	4	4
	T	5	5	5	5	5	5	5	5	5	5	5
	T-E	4-27/32	4-13/16	4-25/32	4-23/32	4-11/16	4-19/32	4-1/2	4-13/32	4-5/16	4-7/32	4-1/8
10	S	4-1/2	4-1/2	4-1/2	4-1/2	4-1/2	4-1/2	4-1/2	4-1/2	4-1/2	4-1/2	4-1/2
	T	5-1/2	5-1/2	5-1/2	5-1/2	5-1/2	5-1/2	5-1/2	5-1/2	5-1/2	5-1/2	5-1/2
	T-E	5-11/32	5-5/16	5-9/32	5-7/32	5-3/16	5-3/32	5	4-29/32	4-13/16	4-23/32	4-5/8
11	S	5	5	5	5	5	5	5	5	5	5	5
	T	6	6	6	6	6	6	6	6	6	6	6
	T-E	5-27/32	5-13/16	5-25/32	5-23/32	5-11/16	5-19/32	5-1/2	5-13/32	5-5/16	5-7/32	5-1/8
12	S	6	6	6	6	6	6	6	6	6	6	6
	T	6	6	6	6	6	6	6	6	6	6	6
	T-E	5-27/32	5-13/16	5-25/32	5-23/32	5-11/16	5-19/32	5-1/2	5-13/32	5-5/16	5-7/32	5-1/8

1. Tolerances are specified in ANSI/ASME B18.2.1. Full-body diameter and reduced body diameter lag screws are shown. For reduced body diameter lag screws, the unthreaded body diameter may be reduced to approximately the root diameter, D_r.
2. Minimum thread length (T) for lag screw lengths (L) is 6 or 1/2 the lag screw length plus 0.5, whichever is less. Thread lengths may exceed these minimums up to the full lag screw length (L).

Table L3 Standard Wood Screws[1,6]



Cut Thread[2]



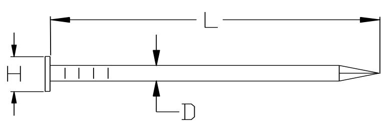
Rolled Thread[3]

D = diameter, in.
D_H = head diameter[5], in.
D_r = root diameter, in.
L = screw length, in.
T = thread length, in.

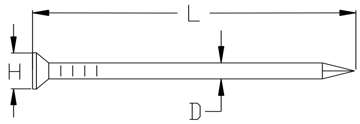
	Wood Screw Number										
	6	7	8	9	10	12	14	16	18	20	24
D	0.138	0.151	0.164	0.177	0.19	0.216	0.242	0.268	0.294	0.32	0.372
D_r[4]	0.113	0.122	0.131	0.142	0.152	0.171	0.196	0.209	0.232	0.255	0.298
D_H[5]	0.262	0.287	0.312	0.337	0.363	0.414	0.480	0.515	0.602	0.616	0.724

1. Tolerances specified in ANSI/ASME B18.6.1.
2. Thread length on cut thread wood screws is approximately 2/3 of the wood screw length, L.
3. Single lead thread shown. Thread length is at least four times the screw diameter or 2/3 of the wood screw length, whichever is greater. Wood screws which are too short to accommodate the minimum thread length, have threads extending as close to the underside of the head as practicable.
4. Taken as the average of the specified maximum and minimum limits for body diameter of rolled thread wood screws.
5. Taken as the average of the specified maximum and minimum limits for head diameter.
6. It is permitted to assume the length of the tapered tip is 2D.

Table L4 Standard Common, Box, and Sinker Steel Wire Nails[1,2]



Common or Box



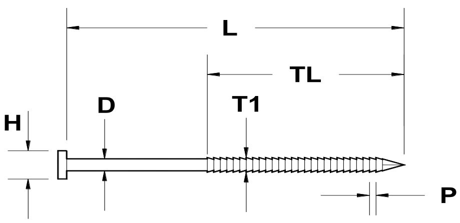
Sinker

D = diameter, in.
L = length, in.
H = head diameter, in.

Type		Pennyweight										
		6d	7d	8d	10d	12d	16d	20d	30d	40d	50d	60d
Common	L	2	2-1/4	2-1/2	3	3-1/4	3-1/2	4	4-1/2	5	5-1/2	6
	D	0.113	0.113	0.131	0.148	0.148	0.162	0.192	0.207	0.225	0.244	0.263
	H	0.266	0.266	0.281	0.312	0.312	0.344	0.406	0.438	0.469	0.5	0.531
Box	L	2	2-1/4	2-1/2	3	3-1/4	3-1/2	4	4-1/2	5		
	D	0.099	0.099	0.113	0.128	0.128	0.135	0.148	0.148	0.162		
	H	0.266	0.266	0.297	0.312	0.312	0.344	0.375	0.375	0.406		
Sinker	L	1-7/8	2-1/8	2-3/8	2-7/8	3-1/8	3-1/4	3-3/4	4-1/4	4-3/4		5-3/4
	D	0.092	0.099	0.113	0.12	0.135	0.148	0.177	0.192	0.207		0.244
	H	0.234	0.250	0.266	0.281	0.312	0.344	0.375	0.406	0.438		0.5

1. Tolerances are specified in ASTM F1667. Typical shape of common, box, and sinker steel wire nails shown. See ASTM F 1667 for other nail types.
2. It is permitted to assume the length of the tapered tip is 2D.

Table L5 Post-Frame Ring Shank Nails[1]

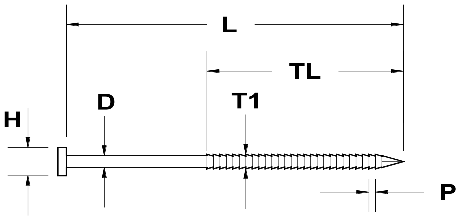


D = diameter, in.
L = length, in.
H = head diameter, in.
TL = minimum length of threaded shank, in.
T1 = crest diameter, in.
$D + 0.005 \text{ in.} \leq T1 \leq D + 0.010 \text{ in.}$
P = pitch or spacing of threads, in.
$0.05 \text{ in.} \leq P \leq 0.077 \text{ in.}$

D	L	TL	H	Root Diameter[2], D_r
0.135	3, 3.5	2.25	5/16	0.128
0.148	3, 3.5, 4	2.25	5/16	0.140
	4.5	3		
0.177	3, 3.5, 4	2.25	3/8	0.169
	4.5, 5, 6, 8	3		
0.200	3.5, 4	2.25	15/32	0.193
	4.5, 5, 6, 8	3		
0.207	4	2.25	15/32	0.199
	4.5, 5, 6, 8	3		

1. Tolerances are specified in ASTM F1667.
2. Root diameter is a calculated value and is not specified as a dimension to be measured.

Table L6 Roof Sheathing Ring Shank Nails[1]

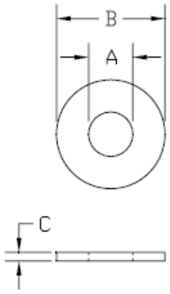


D = diameter, in.
L = length, in.
H = head diameter, in.
TL = minimum length of threaded shank, in.
T1 = crest diameter, in.
$D + 0.005 \text{ in.} \leq T1 \leq D + 0.012 \text{ in.}$
P = pitch or spacing of threads, in.
$0.05 \text{ in.} \leq P \leq 0.077 \text{ in.}$

Dash No.	D	L	TL	H
01	0.113	2-3/8	1-1/2	0.281
02	0.120	2-1/2	1-1/2	0.281
03	0.131	2-1/2	1-1/2	0.281
04	0.120	3	1-1/2	0.281
05	0.131	3	1-1/2	0.281

1. Tolerances are specified in ASTM F1667.

Table L7 Standard Cut Washers[1]



A = inside diameter, in.

B = outside diameter, in.

C = thickness, in.

Nominal Washer Size	A Inside Diameter Basic	B Outside Diameter Basic	C Thickness Basic
3/8	0.438	1.000	0.083
1/2	0.562	1.375	0.109
5/8	0.688	1.750	0.134
3/4	0.812	2.000	0.148
7/8	0.938	2.250	0.165
1	1.062	2.500	0.165

1. Tolerances are provided in ANSI/ASME B18.22.1. For other standard cut washers, see ANSI/ASME B18.22.1.

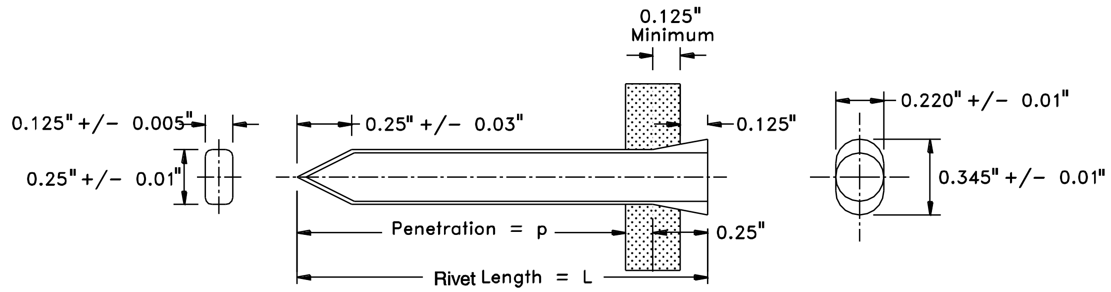
A

APPENDIX

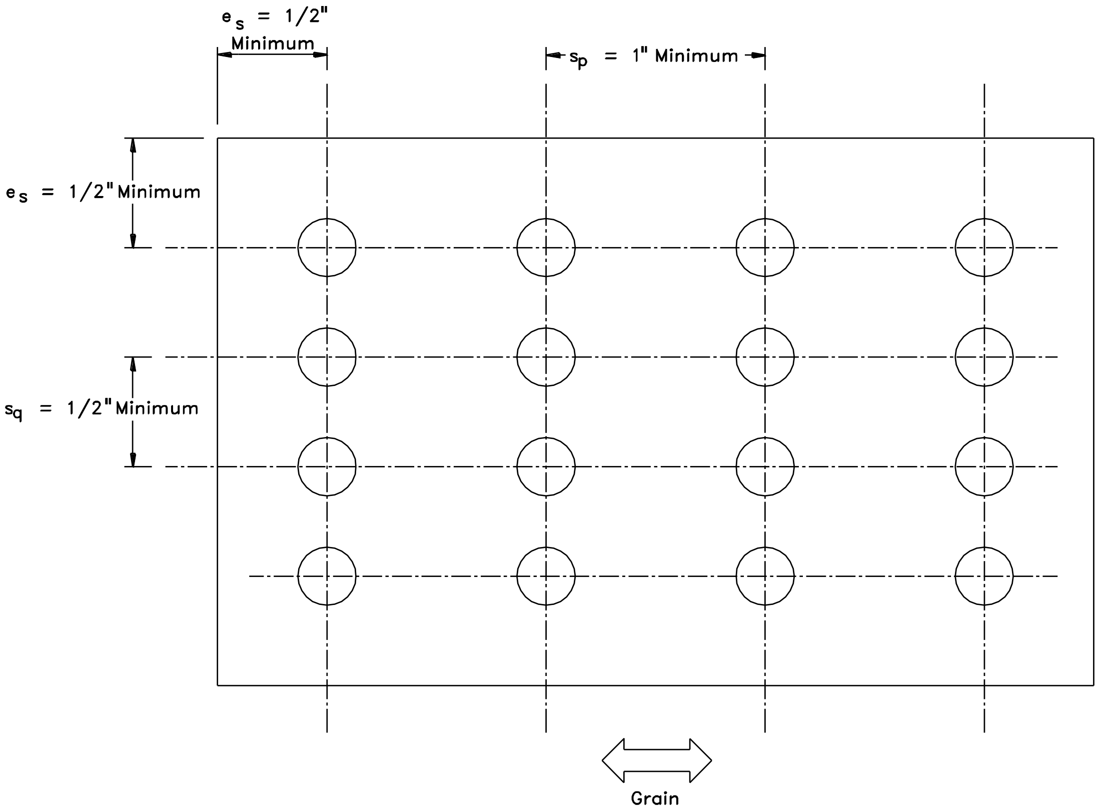
Appendix M (Non-mandatory) Manufacturing Tolerances for Rivets and Steel Side Plates for Timber Rivet Connections

Rivet dimensions are taken from ASTM F 1667.

Rivet Dimensions



Steel Side Plate Dimensions



Notes:

1. Hole diameter: 17/64 minimum to 18/64 maximum.
2. Tolerances in location of holes: 1/8 maximum in any direction.
3. All dimensions are prior to galvanizing in inches.
4. s_p and s_q are defined in 14.3.
5. e_s is the end and edge distance as defined by the steel.
6. Orient wide face of rivets parallel to grain, regardless of plate orientation.

Appendix N (Mandatory) Load and Resistance Factor Design (LRFD)

N.1 General

N.1.1 Application

LRFD designs shall be made in accordance with Appendix N and all applicable provisions of this Specification. Applicable loads and load combinations, and adjustment of design values unique to LRFD are specified herein.

N.1.2 Loads and Load Combinations

Nominal loads and load combinations shall be those required by the applicable building code. In the absence of a governing building code, the nominal loads and associated load combinations shall be those specified in ASCE 7.

N.2 Design Values

N.2.1 Design Values

Adjusted LRFD design values for members and connections shall be determined in accordance with ASTM Specification D 5457 and design provisions in this Specification or in accordance with N.2.2 and N.2.3. Where LRFD design values are determined by the reliability normalization factor method in ASTM D 5457, the format conversion factor shall not apply (see N.3.1).

N.2.2 Member Design Values

Reference member design values in this Specification shall be adjusted in accordance with 4.3, 5.3, 6.3, 7.3, 8.3, 9.3, and 10.3 for sawn lumber, structural glued laminated timber, poles and piles, prefabricated wood I-joists, structural composite lumber, panel products, and cross-laminated timber, respectively, to determine the adjusted LRFD design value.

N.2.3 Connection Design Values

Reference connection design values in this Specification shall be adjusted in accordance with Table 11.3.1 to determine the adjusted LRFD design value.

N.3 Adjustment of Reference Design Values

N.3.1 Format Conversion Factor, K_F (LRFD Only)

Reference design values shall be multiplied by the format conversion factor, K_F, as specified in Table N1. Format conversion factors in Table N1 adjust reference ASD design values (based on normal duration) to the LRFD reference resistances (see Reference 55). Format conversion factors shall not apply where LRFD reference resistances are determined in accordance with the reliability normalization factor method in ASTM D 5457.

Table N1 Format Conversion Factor, K_F (LRFD Only)

Application	Property	K_F
Member	F_b	2.54
	F_t	2.70
	F_v, F_{rt} F_s	2.88
	F_c,	2.40
	$F_{c\perp}$	1.67
	E_{min}	1.76
All Connections	(all design values)	3.32

N.3.2 Resistance Factor, ϕ (LRFD Only)

Reference design values shall be multiplied by the resistance factor, ϕ, as specified in Table N2 (see Reference 55).

Table N2 Resistance Factor, ϕ (LRFD Only)

Application	Property	Symbol	Value
Member	F_b	ϕ_b	0.85
	F_t	ϕ_t	0.80
	F_v, F_{rt}, F_s	ϕ_v	0.75
	F_c, $F_{c\perp}$	ϕ_c	0.90
	E_{min}	ϕ_s	0.85
All Connections	(all design values)	ϕ_z	0.65

N.3.3 Time Effect Factor, λ (LRFD Only)

Reference design values shall be multiplied by the time effect factor, λ, as specified in Table N3.

Table N3 Time Effect Factor, λ (LRFD Only)

Load Combination[2]	λ
1.4D	0.6
1.2D + 1.6L + 0.5(L_r or S or R)	0.7 when L is from storage 0.8 when L is from occupancy 1.25 when L is from impact[1]
1.2D + 1.6(L_r or S or R) + (L or 0.5W)	0.8
1.2D + 1.0W + L + 0.5(L_r or S or R)	1.0
1.2D + 1.0E + L + 0.2S	1.0
0.9D + 1.0W	1.0
0.9D + 1.0E	1.0

1. Time effect factors, λ, greater than 1.0 shall not apply to connections or to structural members pressure-treated with water-borne preservatives (see Reference 30) or fire retardant chemicals.
2. Load combinations and load factors consistent with ASCE 7-16 are listed for ease of reference. Nominal loads shall be in accordance with N.1.2. D = dead load; L = live load; L_r = roof live load; S = snow load; R = rain load; W = wind load; and E = earthquake load.